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TEMPORARY SANITARY SEWAGE DETENTION SYSTEM

Alberta
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Innovative Housing Grants Program

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TEMPORARY SANITARY SEWAGE DETENTION SYSTEM

March 1988

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The views and conclusions expressed and the recommendations made in this report are entirely those of the authors and should not be construed as expressing the opinions of Alberta Municipal Affairs.

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FOREWORD

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EXECUTIVE SUMMARY

Introduction

The purpose of this study was to determine the feasibility of temporarily storing sanitary sewage flows from a 20 acre subdivision proposed for the Lake District in the City of Edmonton. The permanent servicing system for this 20 acre development would not be in place for a number of years. To allow development to proceed, this report was necessary to develop the parameters to establish a Sanitary Sewage Detention System that would store sanitary flows during storm events when the downstream system would not be able to accommodate these flows.

Study Methods

This report reviewed and correlated all as-built information available for the existing downstream sanitary basin that would be receiving the sanitary flows. Design parameters presently utilized by the City of Edmonton were applied to the collected as-built information to arrive at various design flows. These flows along with measured flows along the system (obtained by the City of Edmonton) enabled Walker, Newby & Associates Ltd. to review the receiving system under actual and design conditions including storm conditions.

A determination was made of the flow that would result in a maximum 12 inch surcharge anywhere in the receiving system. This was considered an acceptable surcharge to the existing system as it would not cause any flooding of basements.

A unit hydrograph for wet weather flow conditions was developed from information supplied from the City of Edmonton and from this hydrograph it was determined that the flow in the existing system would be greater than the 12 inch surcharge flow for a maximum period of 4.1 hours.

Since the downstream system could accept the flow from the proposed 20 acre development except for a period of 4.1 hours during a major storm, the storage facility was sized to provide for 4.5 hours of

storage. The detailed engineering drawings and contract documents for the Lake District sanitary detention system have been prepared and tenders will be called when the development of the subdivision proceeds.

Findings

The storage concept for the Lake District 20 acre development that was examined in this report is considered to be economically feasible. Also considered economically feasible is the extension of the concept to allow for significant increase in the size of existing sanitary drainage basins where the capacity of the existing collection system is, at present, apparently fully utilized.

The detention concept could also be used within existing developments to ameliorate surcharging conditions in the sanitary sewer system.

Conclusions and Recommendations

The results of this study could benefit Albertans by providing a method of extending the life and capacities of sewage transmission and treatment facilities, thus effecting economies in local authorities' servicing budgets and opening up, for development, areas that were previously considered undevelopable because of downstream constraints in sewage systems.

Specific recommendations include:

.1 that the storage facility proposed for the 20 acre development in the Lake District be constucted in conjunction with the subdivision;

.2 that the Lake District storage facility be operated as a pilot project, so that operating, maintenance and cost information can be documented and utilized for the planning of future facilities;

.3 that concepts developed herein, for expanding the size of drainage basins, be applied to locations in Alberta where such expansion may be beneficial.

1.0 INTRODUCTION

1.1 The Problem

.1 General

Sanitary sewage collection systems in some municipalities in Alberta are apparently undersized. This problem is caused by:

a) Increase in Contributing Population

Where a sanitary sewerage system in a municipality has been sized for a particular size of drainage basin and the infrastructure is already in place, problems may be encountered when the basin is increased in size to accommodate expansion of the municipality. The population increase may be inhibited or new trunk lines may have to be constructed at great expense, for relatively long distances, to connect the new population to the point of sewage treatment or system discharge.

b) Overloading of the System

Traditionally, for residential service, sanitary sewers have been designed to conduct the flow from domestic uses such as toilets, sinks, bathtubs, showers, washing machines, dishwashers, etc. The pipes are sized and sloped to conduct the maximum or "peak" sanitary sewage flow, which is normally experienced about breakfast time on a weekday. An additional flow allowance, of 10% in some cases, has been allowed in order to accommodate ground water infiltration into the system from leaking pipe joints or flow from weeping tile. Weeping tile is a system of perforated pipes installed around a house, at bottom of basement level, to conduct ground water away from the basement to prevent dampness or flooding inside the basement of the home. In Alberta, the weeping tile system is normally connected to the sanitary sewage collection system. In recent times, problems arising from insurance related law suits have created a civic awareness that

rainstorms can cause serious flooding in sanitary sewers to the extent that back-up of sanitary sewage has been experienced in residential house basements. Rainstorms can cause a 200% to 300% increase in flow in sanitary sewers rather than the 10% increase that may have been allowed for in the initial design. The large storm inflow comes to the sanitary system, mainly, by way of the weeping tile. Inadequate lot grading can cause storm water to flow towards the house and percolate through poorly compacted basement backfill to the weeping tile system and thus introduce storm flow to the sanitary sewers. Nowadays, some municipal authorities are wary of adding new subdivisions to sanitary systems that have experienced flooding in the past for fear of increasing their liability in the event of future flooding. So, here again, population increase may be inhibited, or large expenditures may be experienced in duplicating the infrastructure system.

.2 Specific

Inconnu Holdings Ltd., a land development company, desires to develop, as a residential subdivision, some 20 acres of land in the Lake District, Neighbourhood 7 in Edmonton, Alberta. The sanitary sewerage servicing scheme for the Lake District Neighbourhood 7 depends on the construction of a sanitary sewage lift station and some five miles of sanitary sewer outfall through presently undeveloped land in the Lake District and Pilot Sound as shown on Figure No. 1 on page 3. The construction contract for this work was tendered in 1984, but the work did not proceed because of fiscal difficulties among the owners. For Inconnu Holdings to construct the sanitary sub-trunks, lift station and outfall necessary to service their proposed subdivision according to approved concepts it would have to spend $4,000,000 for the offsite sanitary trunks for this 20 acre development.

Figure 1

EXISTING SANITARY SERVICING CONCEPT

1.2 Project Objectives

The objective of this project is to solve the specific problem faced by Inconnu Holdings, and by the solving of that problem, to provide a means of mitigating the problems of undersized sanitary sewage collection systems where they occur in other Alberta municipalities.

1.3 Scope and Focus of the Work

.1 To gather base information on flows and pipe capacities in the sanitary sewerage system servicing central north Edmonton adjacent to 20 acres of land in the Lake District owned by Inconnu Holdings Ltd.

.2 To carry out engineering analysis and conceptual designs of a sanitary storage facility for the Inconnu development which would control flow and allow discharge into the existing sanitary sewerage system in central north Edmonton.

.3 To prepare detailed engineering drawings and specifications for all structural, mechanical, electrical and municipal work.

.4 To assess the proposed system in terms of capital and operating costs and compare it with conventional techniques.

.5 To investigate the feasibility of large scale systems in terms of capital and operating costs and downstream sewer capacity.

1.4 Organization of the Report

A major part of the report is the analysis of the existing sanitary sewerage system in central north Edmonton. A reconciliation between theoretical values and monitored values is

carried out. A relationship between rainfall and sanitary sewage flow is identified. The report is technical in nature, with references being made to various explanatory or illustrative tables or figures as required.

1.5 Previous Related Investigations

Previous reports on the subject of Storage of Sanitary Sewage or Sewage Equalization have referred mainly to averaging of flows to increase the efficiency of sewage treatment plants. The 1982 Associated Engineering Services Ltd. report on "Flow Equalization" referred to the optimization of pumping systems in the Province of Alberta. The 1982 Walker, Newby & Associates Ltd. report on "Dunluce/Warwick - N.W. Triangle Sewer Management" identified a relationship between cost viability and rate of development. The Associated Engineering report stated that major peaks in sanitary sewer flows are caused by storm water and that fact will be confirmed and emphasized by this report on the detention system to facilitate development in Lake District Neighbourhood 7. This report, unlike most previous reports, will be specific and will investigate the capacity of the downstream system to accommodate flow from a storage facility.

1.6 Sources and Methods

Information on population, existing and future sanitary systems, monitored results of sanitary sewage flow and rainfall records were obtained from the Drainage Department, City of Edmonton Water & Sanitation. The information is analysed by graphically representing the existing and proposed systems and relating them to future design events.

1.7 Proposed Treatment of Findings

It is proposed that the detailed design produced by this report will be used for construction of a storage facility to be incorporated into a proposed 20 acre subdivision in the Lake District, Neighborhood 7 in Edmonton.

It is also proposed that the results of this report will be of use in the planning and design of similar facilities elsewhere in Alberta and that operating and maintenance information resulting from the operation of the pilot facility in the Lake District Neighbourhood 7 will assist in the implementation of future projects.

1.8 System of Units

Basic flow information, pertaining to the sanitary sewer system, gathered by the City of Edmonton is in imperial units as is the as-built information extracted from City records. To avoid confusion, this report therefore utilizes the imperial system of units instead of the metric system.

2.0 BASE INFORMATION

2.1 Literature Search

The search for information concerning sewage storage involved a review of most of the engineering periodicals in both the Department of Environment (Alberta) library and the Cameron Library of the University of Alberta. Most of the technical reports on this subject are analyses of combined sewer overflow facilities and appear in such periodicals as the Water Pollution Control Federation Journal, Progress in Water Technology and the U.S.A. Environmental Protection Agency reports. Also reviewed was the report entitled "Flow Equalization", a report prepared by Associated Engineering Services Ltd. for Alberta Environment.

A summary of the literature search is as follows:

.1 No relationship has been established between amount of aeration required with respect to time of retention.

.2 Little mention was made of the methods of reducing settling and thus septicity, e.g. keeping the retained fluid in motion.

.3 Cost comparisons were theoretical and tended to be oversimplified.

.4 There is a lack of information on design and operating criteria for equalization facilities remote from treatment plants.

.5 With respect to flows not affected by storm water peaks, equalization does not appear to be a worthwhile tool for smoothing the quality of sewage at treatment plants.

.6 Non-uniformity of flow in tanks could increase retention time for some pockets of sewage beyond the nominally allotted time.

.7 Some information is available on problems actually experienced in operating equalization basins.

.8 Major peaks in sanitary sewer flows are caused by storm water.

.9 In some cases, savings in lift station costs can be achieved by installing an equalization basin upstream of the station.

2.2 Capacity of Existing System

A search was made of City of Edmonton records to determine the size and slope of the existing sanitary sewer line, the "connector collector", see Figure 2, Page 9, which would connect the proposed 20 acre development in the Lake District to a 60" diameter combined storm/sanitary sewer at 127 Avenue and 68 Street. It is assumed that the 60" diameter sewer will easily accommodate the increased flow from the proposed development in Lake District No. 7 since the increased flow would be in the order of 0.10 cubic feet per second being added to a flow of 800 cubic feet per second. The size and slope of the sewer between each manhole was identified and the capacity was determined by the equation,

$$Q = \frac{1.486}{n} \times R^{2/3} \times S^{1/2} \times a$$

Where:

Q = capacity in cubic feet/sec.(c.f.s.)

n = Manning's coefficient of friction normally taken as 0.013 for concrete pipe

R = hydraulic radius, being the cross-sectional area divided by the wetted perimeter in feet

S = slope, feet per foot

a = cross-sectional of area of pipe in square feet

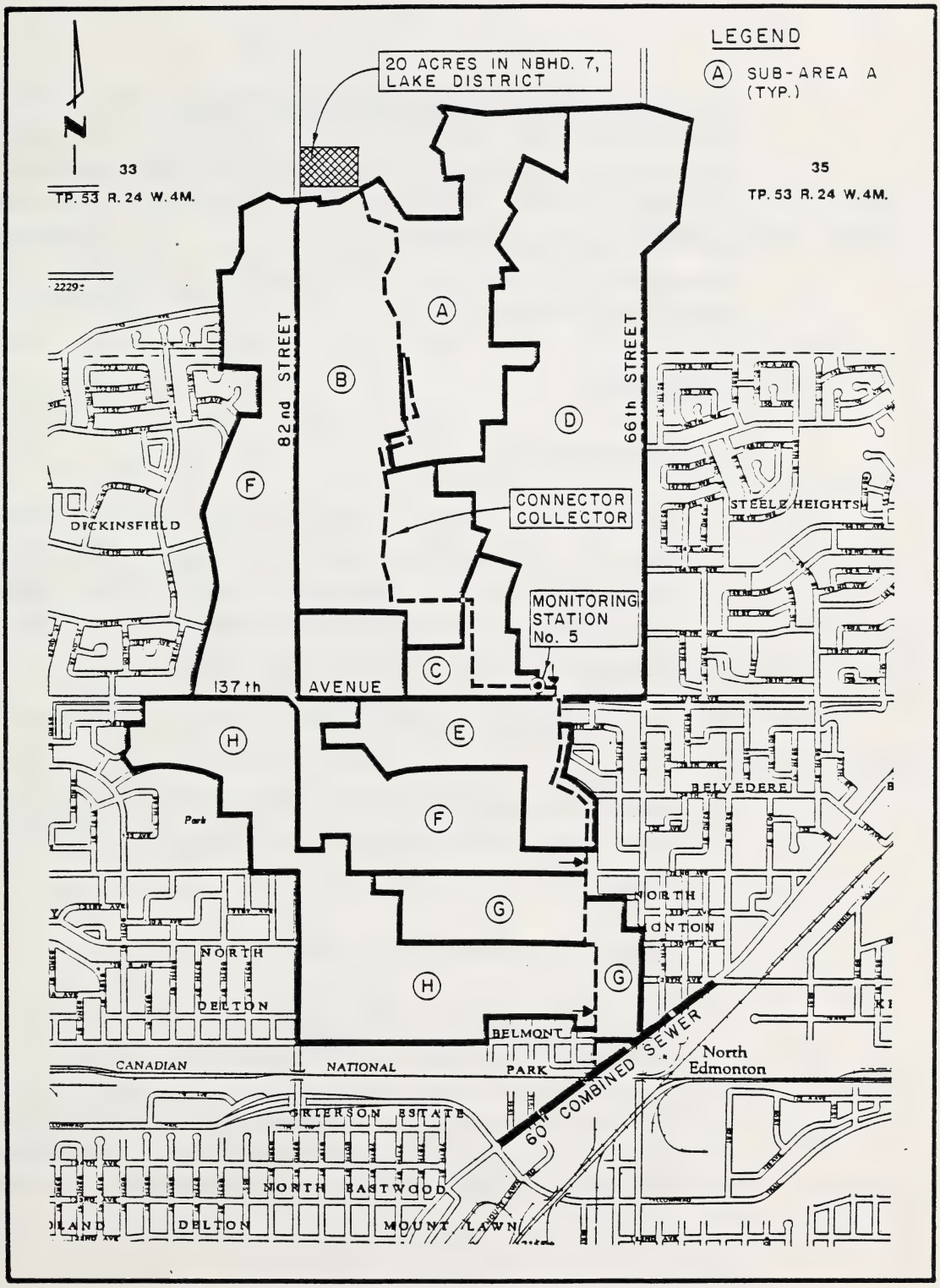


Figure 2 CONNECTOR COLLECTOR DRAINAGE BASIN

The slope is normally determined by dividing the vertical drop between manholes by the distance between manholes. This slope was modified in the case of the connector collector immediately downstream of the Monitoring Station, but this will be discussed later in this report.
The capacity for each section of sewer was calculated and graphically presented.

2.3 Design Flows

.1 Dry Weather Flow

The design dry weather flows were calculated at various points on the downstream connector collector. The design dry weather flow at any point on the sanitary sewer system is a function of the population contributing to that point. The peak flow is represented by this formula.

= Population x Average per capita flow x Harman's Peaking Factor

Average per capita/day flow is taken as 77 imperial gallons per day and Harman's Peaking Factor is represented by

$$\frac{14}{4 + p^{1/2}} + 1$$

where p is the population in thousands.

Data on population for each of the sub-areas draining to the connector collector is shown in Table 1, page 11.

The connector collector runs through some of the sub-areas, A,B,C,E & G while other sub-areas D,F and H discharge to the collector sewer at one point. (See Figure 2, on page 9.)

TABLE 1 - CONNECTOR COLLECTOR DRAINAGE BASIN DATA

SUB AREA DESIGNATION	POPULATION	AREA (ha.)	PIPE mm X km	MANHOLES	SAG MANHOLES	# OF WEEPING TILE CONNECT
A (Existing)	1,900	47.1	1,006.3	92	14	567
Balance of A to Ult.	2,100	55.7	1,219.7	57	6	365
B	3,604	77.3	1,339.1	85	6	499
C	645	17.3	723.2	28	7	220
D (Existing)	3,382	120.5	2,609.5	174	23	1,094
Balance of D to Ult.	2,500	69.5	1,505	100	13	631
E	1,586	43.4	939.8	63	8	394
F	3,845	142.5	3,085.7	206	27	1,293
G	1,579	44.7	968.6	65	9	406
H	2,759	116.0	2,511.9	168	22	1,053

Population data was taken from the 1985 Census results. These population figures were applied to 1986 design calculations since it was figured that little population change had taken place over a year except for the north part of Sub-Area A. The Census Tracts did not correspond with the drainage basin boundaries, so some estimations had to be made using areas and number of housing units as a guide. For Sub-areas A,B & C, upstream from Monitoring Station No. 5, designated as M.S 5 on Figure 2, page 9, at 137 Avenue and 70 Street, a detailed inventory was made of the number and type of housing units and weeping tile connections, the number of manholes, sag manholes, and the lengths of pipe of different diameters. The number of weeping tile connections was computed as being the number of single family units plus one third the number of multi - family units. Areas were computed by planimeter. This data is presented in Table 1, page 11. Pipe, manhole and weeping tile data for sub areas D,E,F,G, & H and for future expansions to areas A and D were calculated by extrapolating from the values determined for Sub-Areas A,B,& C. Table 2 on page 13 shows the results of the calculations of dry weather flow and the additional wet weather flow which will be discussed later.

The "ultimate dry weather flow" is the future flow when the total basin is developed and is computed by adding forecasted data from those parts of Sub-Areas A and D which are not yet developed.

.2 Wet Weather Flows

The wet weather flow in a sanitary sewer is defined as the increased rate of flow caused by rainstorm activity. As mentioned previously, the number of weeping tile connections, manholes, sag manholes and lengths of pipe of different diameters was inventoried or extrapolated with the results, for each sub-area, being shown on Table 1, page 11. A sag manhole is defined as a sanitary sewer manhole located at a low point in a street such that if the storm sewer system were to be unable to drain away a heavy rainstorm water would pond in the area and inundate the cover of the sanitary sewer manhole.

TABLE NO. 2
CONNECTOR COLLECTOR - COMPUTATION OF
DRY AND WET WEATHER FLOW

DESIGN DRY-WEATHER FLOW (cfs)		DESIGN WET-WEATHER FLOW (cfs)					
JULY 1986	ULTIMATE	JULY 1986			ULTIMATE		
A. Pop. 1,900	Pop. 4,000			Cumula	Infiltration	.02	Cumula
A. Pop. 1,900	Pop. 4,000	Infiltration	.01	tive			tive
		Manholes	.16		Manholes	.26	
Q = 0.98 cfs	Q=1.91 cfs	Sag MHS	.50		Sag MHS	.71	
		Weeping Tile	1.20		Weeping Tile	1.97	
			1.87	1.87		2.96	2.96
B. Pop. 3,604	Pop. 7,604	Infiltration	.01		Infiltration		
		Manholes	.15		Manholes		
		Sag MHS	.21		Sag MHS		
Q = 2.54 cfs	Q = 3.35 cfs	Weeping Tile	1.06		Weeping Tile		
			1.43	3.30		1.43	4.39
C. Pop. 6,149	Pop. 8,249	Infiltration	.005		Infiltration		
		Manholes	.05		Manholes		
		Sag MHS	.245		Sag MHS		
Q = 2.79 cfs	Q = 3.60 cfs	Weeping Tile	.47		Weeping Tile		
			0.77	4.07		0.77	5.16
D. Pop. 9,531	Pop. 14,131	Infiltration	.02		Infiltration	.03	
		Manholes	.31		Manholes	.49	
		Sag MHS	.81		Sag MHS	1.27	
Q = 4.08 cfs	Q = 5.68 cfs	Weeping Tile	2.34		Weeping Tile	3.69	
			3.48	7.55		5.48	10.64
E. Pop. 11,117	Pop. 15,717	Infiltration	.01		Infiltration		
		Manholes	.11		Manholes		
		Sag MHS	.28		Sag MHS		
Q = 4.64 cfs	Q = 6.22 cfs	Weeping Tile	.84		Weeping Tile		
			1.24	8.79		1.24	11.88
F. Pop. 14,962	Pop. 19,562	Infiltration	.03		Infiltration		
		Manholes	.36		Manholes		
		Sag MHS	.96		Sag MHS		
Q = 5.96 cfs	Q = 7.47 cfs	Weeping Tile	2.75		Weeping Tile		
			4.10	12.89		4.10	15.98
G. Pop. 16,541	Pop. 21,141	Infiltration	.01		Infiltration		
		Manholes	.11		Manholes		
		Sag MHS	.32		Sag MHS		
Q = 6.49 cfs	Q = 7.97 cfs	Weeping Tile	.86		Weeping Tile		
			1.30	14.19		1.30	17.28
H. Pop. 19,300	Pop. 23,900	Infiltration	.02		Infiltration		
		Manholes	.29		Manholes		
		Sag MHS	.78		Sag MHS		
Q = 7.39 cfs	Q = 8.23 cfs	Weeping Tile	2.24		Weeping Tile		
			3.33	17.52		3.33	20.61

Wet weather flows were calculated by the following parameters which have been established by the City of Edmonton, Water and Sanitation Drainage Branch.

.1 Infiltration allowance of 20 litres/day/mm dia/km of sewer pipe. The allowance varies with the type of pipe but, since the values were relatively insignificant, use was made in all cases of the one fairly liberal value of 20 litres/day.

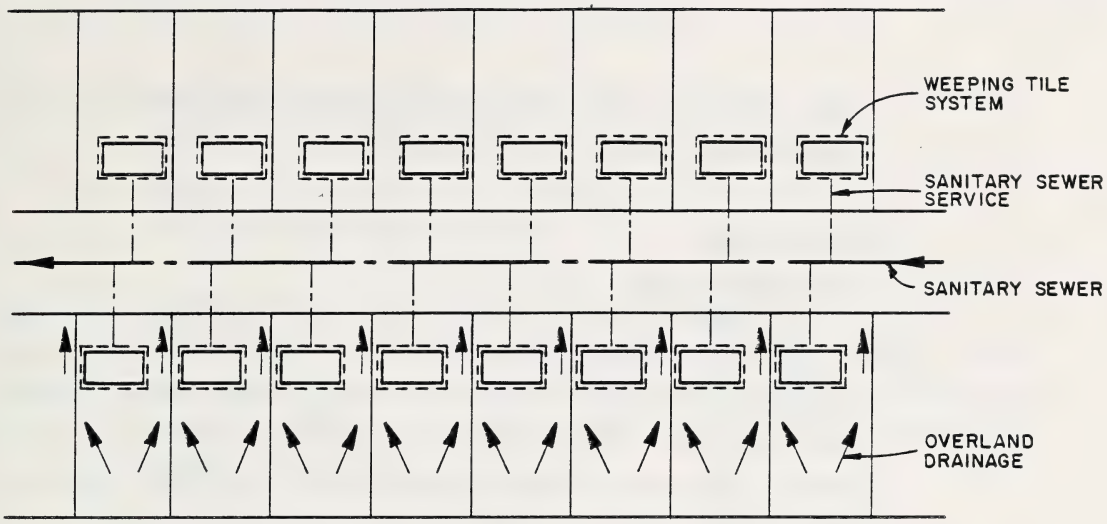
.2 Infiltration allowance of 0.05 L/second for every manhole.

.3 Infiltration allowance of 1.0 L/sec for every sag manhole.

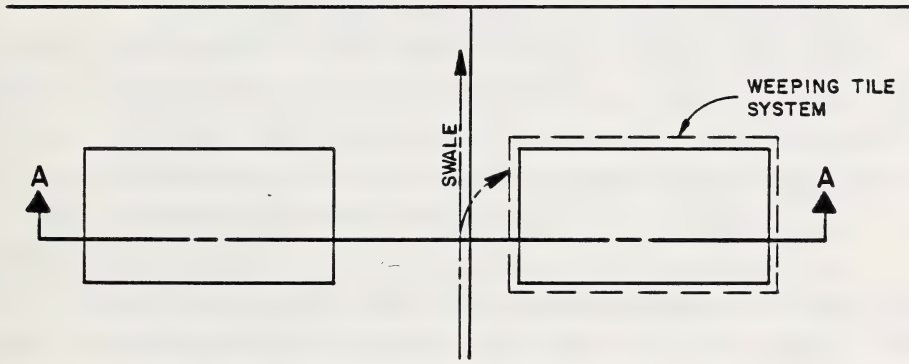
.4 Allowance of 0.06 L/sec for each house for weeping tile infiltration. *

The above allowances were calculated as shown on Table 2, page 13, and are added to the peak dry weather flow to produce the peak wet weather flow. The values of ultimate wet weather flows were graphed. As with dry weather flow, "ultimate" means the addition of the presently undeveloped parts of Sub-areas A and D.

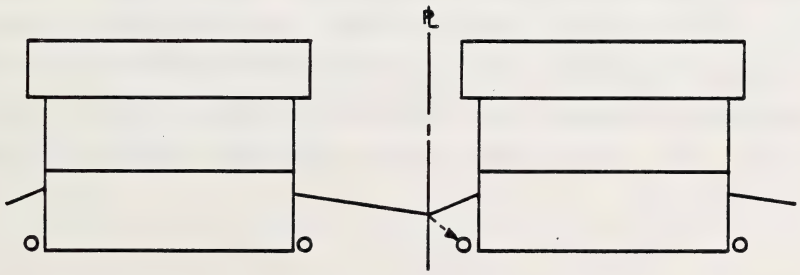
* Refer to Figure 3 on page 15. Normally residential lots slope from back to front and rainfall on the lot is absorbed in the ground or flows towards the front of the lot as shown. The enlarged view shows that the swale between lots is typically closer to one house than the other. In the situation where basement backfill is improperly compacted it may contain voids which would allow surface water to percolate down to the bottom of the basement, the weeping tile area, as shown on the enlarged view section A-A. Or, poor lot grading may direct all surface flow towards the building; and from there a large part of it would enter the weeping tile system. It takes only one lot, poorly graded or poorly compacted, out of the 16 shown in



TYPICAL PLAN



DETAIL



CROSS SECTION A-A

Figure 3

WEEPING TILE EFFECT

Figure No. 3 to produce a weeping tile average flow of 0.06 litres/second during a rainstorm event.

2.4 Flow Monitoring

.1 Monitored Results

Use was made of flow records from a sanitary sewer monitoring station (Monitoring Station No. 5) which was set up and operated by the City of Edmonton Drainage Department at Manhole A9 at 137 Avenue and 70 Street. At the monitoring station, the depth of flow was measured by sonic depth measuring instruments and recorded on 7 day charts. The station is located on the connector collector as shown on Figure 2, page 9. Flow results from July and August 1986 are used for this report.

.2 Correlation Between Calculated and Monitored Flows

Monitored results supplied by the City of Edmonton in terms of cubic feet per second were based on the calibration of flows derived from measurements of velocity and depth of flow in Manhole A9, Monitoring Station No. 5. Calculation of flows for the purposes of this report was carried out using the depth of flow monitored by the city applied to the hydraulic characteristics of the downstream run of sewer to the next manhole. The flows derived by this latter method were significantly lower than the city values but they matched the pipe capacity criteria and also accorded with design flow values. Included in the flow calculations was an allowance for loss of head at the next downstream manhole, Manhole A10. At A10, the connector collector running to the east, joins a north-south line which enlarges at Manhole A10 from an 18" diameter to 24" diameter sewer. The loss of head allowance for this situation was taken as $1.5 \times \frac{V^2}{2g}$

where V is flow velocity in ft/sec. and g is acceleration due to gravity in ft/sec/sec.

The flow calculations were based on a vertical drop of 0.68 ft. over 323 ft. of 18" diameter sewer; with a Manning n = 0.013. Results of the flow calculations for Monitoring Station No. 5 are shown on Table 3, page 18.

24 Hour Graphs of dry weather flow and wet weather flow at M.S. #5 are shown on Figure No. 4, page 19. The wet weather graph was derived from monitored readings at Monitoring Station #5 on July 17, 1986 when surcharging was experienced in the manhole during a rainstorm.

Between the 6th and 24th of August no rain was recorded in the area according to the rain gauge at the Grand Trunk Arena. From that it was decided that 14 August 1986 represented a typical dry weather flow day and thus a graph of this day's flow has been superimposed on the graph of wet weather flow for July 17th as shown on Fig. No. 4. The dry weather flow graph for the 14th of August was subtracted from the wet weather flow graph for July 17th to produce a net wet weather flow graph for July 17th as shown on Figure No. 5, page 20.

The rain record for July 17th from the Grand Trunk Arena Raingauge Station is superimposed on the net wet weather flow graph (same horizontal time scale/different vertical scale). The Grand Trunk Arena Raingauge Station is situated in the north end of the City as is Monitoring Station No. 5 and it is considered that the Grand Trunk Arena rainfall record provides a fair approximation of the rainfall event that caused the increase in sanitary sewer flow at Monitoring Station No. 5.

TABLE NO. 3
MONITORING STATION NO. 5 - SEWAGE FLOWS
18" DIAMETER SEWER LINE
JULY/AUGUST, 1986

NORMAL FLOW				SURCHARGED		
FLOW DEPTH %	FLOW DEPTH FEET	VELOCITY FT/SEC	FLOW CFS	SURCHARGE FEET	VELOCITY FT/SEC.	FLOW CFS
10	0.15	1.07	0.11	0.10	2.65	4.69
20	0.30	1.63	0.40	0.20	2.80	4.95
30	0.45	1.96	0.86	0.30	2.94	5.19
40	0.60	2.18	1.43	0.40	3.07	5.43
50	0.75	2.38	2.10	0.50	3.27	5.78
60	0.90	2.54	2.78	0.60	3.39	6.00
70	1.05	2.56	3.35	0.70	3.57	6.31
80	1.20	2.62	3.93	0.80	3.69	6.52
90	1.35	2.57	4.27	0.90	3.80	6.71
100	1.50	2.38	4.20	1.00	3.91	6.91

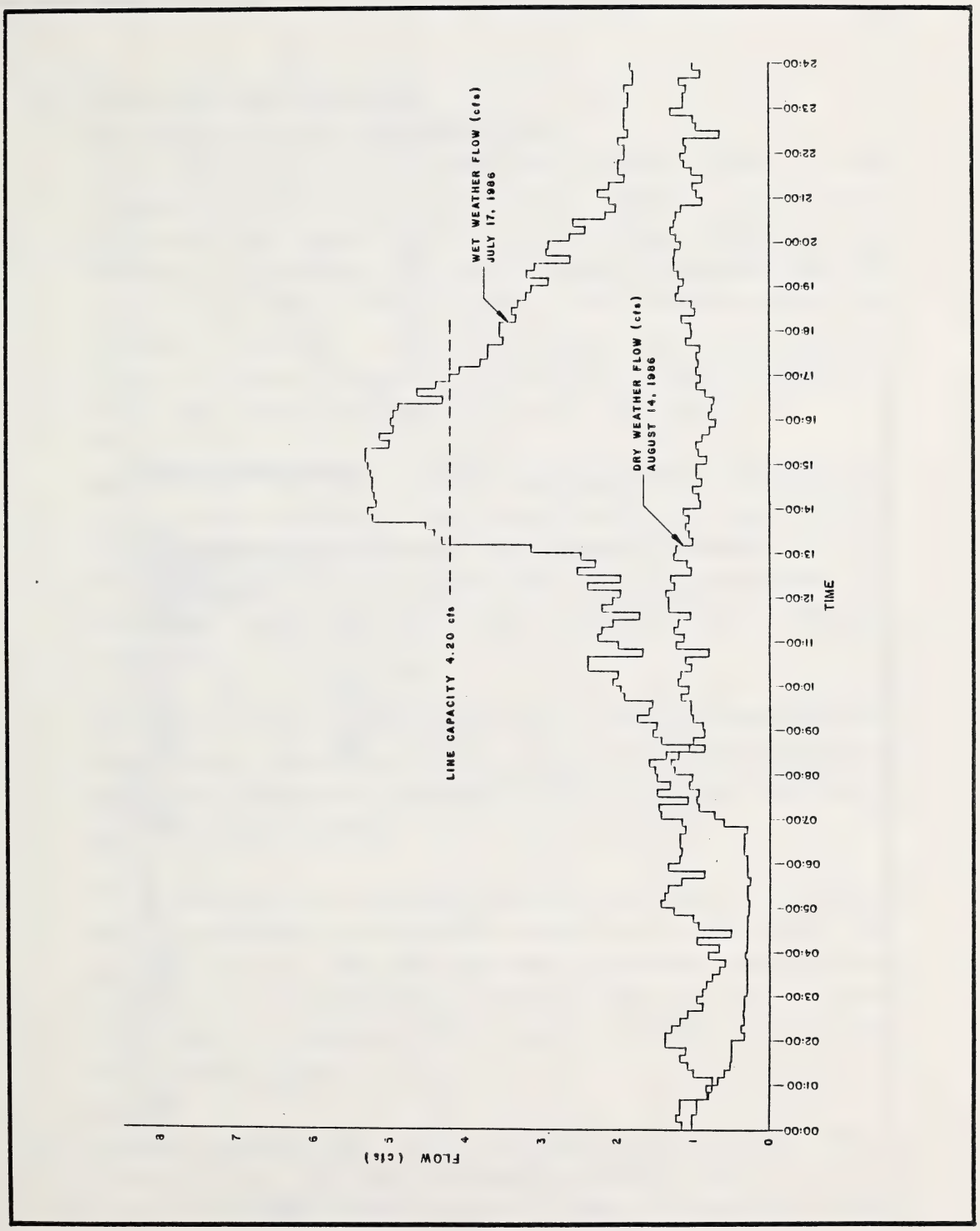


Figure 4

HYDROGRAPHS AT MONITORING STATION No. 5

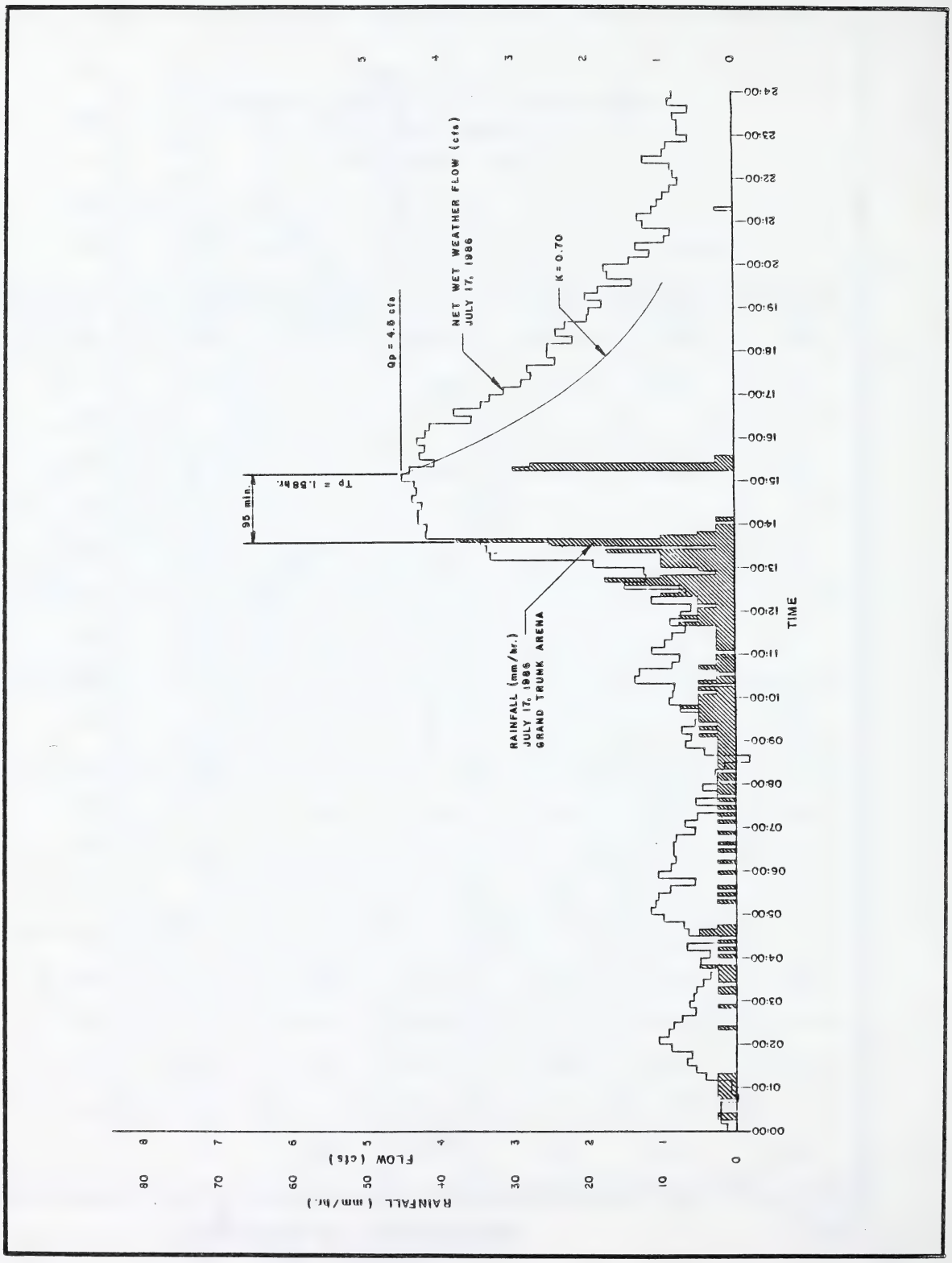


Figure 5

COMBINED GRAPHS; SANITARY HYDROGRAPH AND RAINFALL RECORD FOR JULY 17, 1986 AT MONITORING STATION No. 5

3.0 ANALYSIS AND CONCEPTUAL DESIGN

3.1 Critical Location in Downstream System

The design flows in and capacities of each section of the "Connector Collector" were determined and analysed. As mentioned previously, the "Connector Collector" is the sanitary sewer that is planned as a discharge line for flow from the proposed 20 acre subdivision. Station 0+00 is taken as the upstream end of the "Connector Collector". Monitoring Station No. 5 is located at Station 88+31.51. Connection to the existing 60" diameter combined sewer takes place at Station 153+40 (i.e. 15,340 feet).

Comparison of the design "ultimate dry weather flow" (including the flow from the proposed 20 acre subdivision) and the pipe capacity for each section shows that the pipe capacity is adequate. In the 2,800 feet of sewer main upstream from Monitoring Station No. 5, the design "ultimate dry weather flow" is fairly close to the pipe capacity.

However, the design "ultimate WET weather flow" is greater than the pipe capacity from Station 0+00 to Station 120+00; and the design CURRENT WET weather flow from Station 54+00 to Station 120+00 also surcharges the Collector.

It is interesting to note that the peak storm flow on July 17th caused surcharging at Monitoring Station No. 5, but flow in the sanitary sewer was less than the design current wet weather flow. The storm of July 17th would be classed as being less than a 5 year storm and it is presumed that the manhole infiltration factor did not take full effect.

It is considered that a 12" surcharge in the sanitary sewer collector will not cause flooding in basements since the floor level of a basement should always be at least 12" above the top of the sewer main if the sewer service is installed correctly and the house is properly set with respect to elevation.

Various flows were calculated for the system until a 12" surcharge was evident at one of the manholes. This occured at the simulated flow obtained by adding 32% of the ultimate wet weather flow to the ultimate dry weather flow at a manhole located at Station 7194.86, which is a manhole in a lane east of 74 Street and south of 140 Avenue. Table 4 on page 23 provides the flows and surcharge levels at various manholes for the foregoing calculation.

3.2 Length of Time of Critical Event

.1 The City of Edmonton has reviewed its information from various monitoring stations located around the City. From this preliminary information, two key items have resulted.

a) The City has developed a preliminary graph plotting time for a hydrograph to peak versus the sanitary basin area. We extrapolated this graph for the drainage basin contributing to Monitoring Station No. 5 (480 acres). As shown on Figure 6, page 24 the graph indicates that for an area of 480 acres a typical hydrograph will take 4 hours to peak.

b) The City of Edmonton has also estimated the shape of a unit sanitary hydograph by developing a formula to represent the receding leg of a typical sanitary hydrograph. The formula is:

$$Q = Qpk^{(T-Tp)}$$

Where Qp = Peak Flow
Tp = Time to Peak
k = Receding coefficient
T = Time elapsed after Tp

Through a trial and error procedure the City determined that a k value of 0.70 best represented a typical receding curve for a sanitary hydrograph.

TABLE NO. 4

PROPOSED SURCHARGE LEVELS AND FLOWS

STATION FROM	STATION TO	DIA. DIA.	LENGTH	SLOPE	PIPE CAP.cfs	SURCHARGED FLOW cfs	HEAD (ft) AT UPSTREAM STATION
11923.51	12215.51	24	292	0.0012	7.86	10.23	0.230
11611.51	11923.51	24	312	0.0020	10.14	10.11	0.0194
11405.51	11611.51	24	206	0.0033	13.03	9.99	0.00
No surcharge from Stn. 11405.51 to 10422.51							
10196.51	10422.51	24	226	0.0017	9.35	9.52	0.0087
9986.51	10196.51	24	210	0.0016	9.07	9.44	0.0321
9644.51	9986.51	24	342	0.0023	10.88	9.31	0.00
No surcharge from Stn. 9644.51 to 9154.51							
8831.51	9154.51	18	323	0.0021	4.83	5.32	0.0969
8535.51	8831.51	18	296	0.0011	3.49	5.25	0.464
8181.51	8535.51	18	354	0.0014	3.94	5.20	0.782
7850.51	8181.51	18	331	0.002	4.71	5.14	0.867
7174.51	7850.51	18	176	0.0024	5.16	5.09	0.834
7521.01	7174.51	18	153.5	0.0024	5.16	5.06	0.801
7194.86	7521.01	18	326.15	0.0015	4.08	5.03	1.02
6868.86	7194.86	18	326	0.0028	5.57	4.98	0.799
6760.86	6868.86	18	108	0.0028	5.57	4.92	0.721
6538.86	6760.86	18	222	0.0016	4.21	4.90	0.824
6523.86	6538.86	18	15	0.0017	4.34	4.87	0.829
6397.36	6523.86	18	126.5	0.0029	5.67	4.86	0.179
6097.36	6397.86	18	300	0.0011	3.49	4.84	0.994
5762.36	6097.36	18	335	0.0024	5.16	4.79	0.853
5485.36	5762.36	18	277	0.0018	4.47	4.74	0.891
5157.36	5485.36	18	328	1.02	10.63	4.49	0.00
5091.86	5157.36	18	65.5	5.76	25.27	4.46	0.00
No Further Surcharge Along Connector Collector							

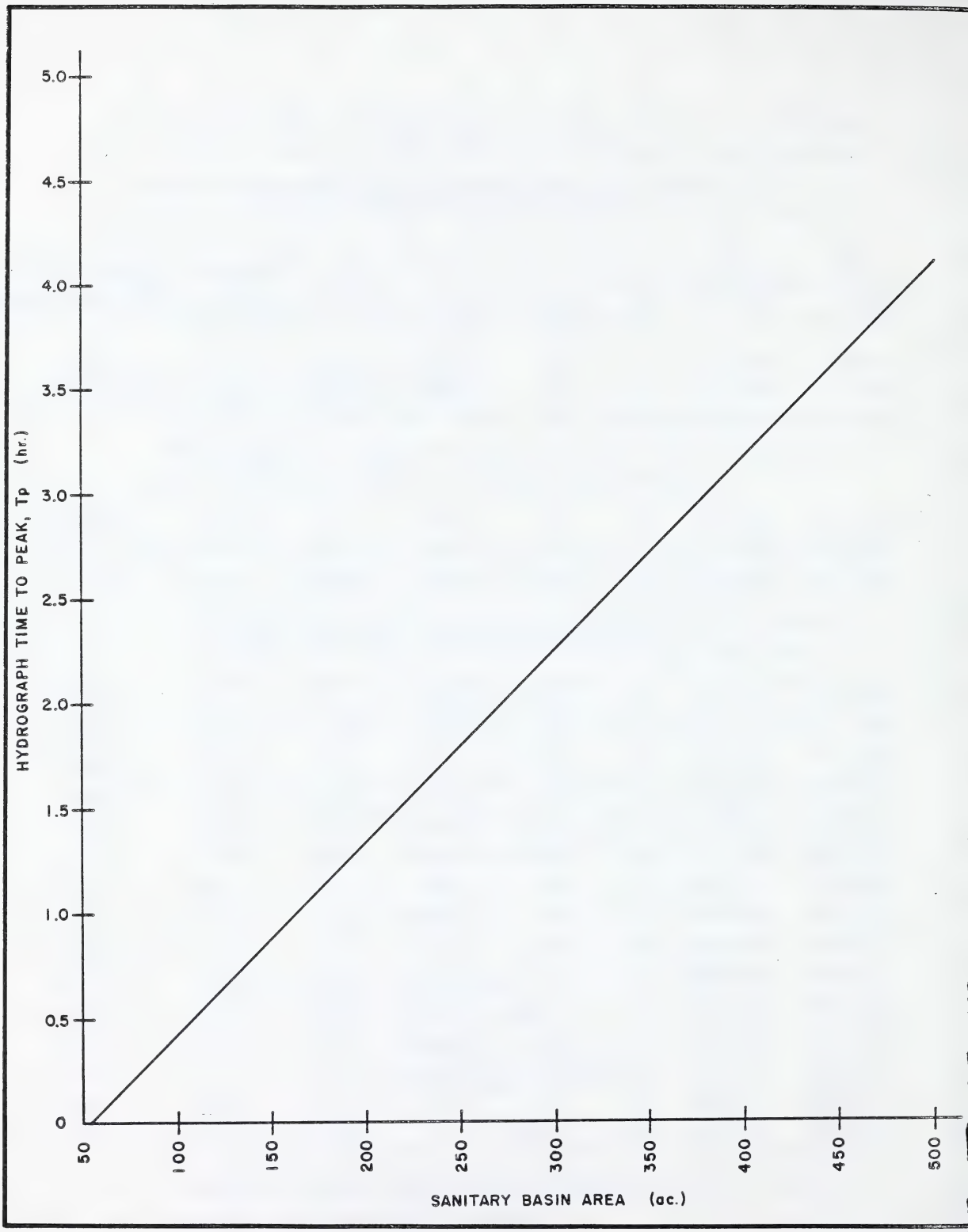


Figure 6

SANITARY HYDROGRAPH;
TIME TO PEAK VS. AREA

.2 As shown on Figure 7 on page 26, by utilizing the two items of information that the City estimated, we have developed a unit sanitary hydrograph for the sanitary basin contributing to Monitoring Station No. 5. The calculated ultimate wet weather flow at Monitoring Station No. 5 is 8.76 c.f.s.; while the calculated peak dry weather flow was determined to be 3.60 c.f.s. (See Table No. 2 for flow from Area C).

We began our hydrograph at 3.6 c.f.s. at Time 0, thereby assuming that the effects of the wet weather flow will coincide with peak dry weather flow. We estimated a straight line to Time to Peak of 4.0 hours (determined from figure 3) with our calculated peak flow of 8.76 c.f.s. occurring at 4.0 hours. The formula derived by the City of Edmonton was then utilized to estimate the shape of the receding curve.

.3 As determined earlier, a flow of 5.25 c.f.s. at Monitoring Station No. 5 results in a surcharge of 12" at an upstream sewer (manhole located in a lane east of 74 Street and south of 140 Avenue). The surcharge calculated at Monitoring Station No. 5 is 1.16 inches and results in the flow of 5.25 c.f.s. It is hereby proposed that when the surcharge at Monitoring Station No. 5 is less than 1.2 inches, the 0.18 c.f.s. peak flow from the proposed 20 acre development in the Lake District, Neighbourhood 7, be permitted to enter the system. Note that the 0.18 c.f.s. flow from the 20 acre development will be dry weather flow since weeping tile systems will not be connected to the sanitary sewer in the new development and manholes will be planned to experience a minimum of infiltration.

AS SHOWN ON FIGURE 7 THE FLOW SHOWN IN THE UNIT HYDROGRAPH EXCEEDS THE 5.25 C.F.S. AT MONITORING STATION NO. 5 FOR A PERIOD OF 4.1 HOURS. THEREFORE, IT SHOULD BE PRUDENT TO TERMINATE THE FLOW FROM THE 20 ACRE DEVELOPMENT FOR A PERIOD OF 4.5 HOURS.

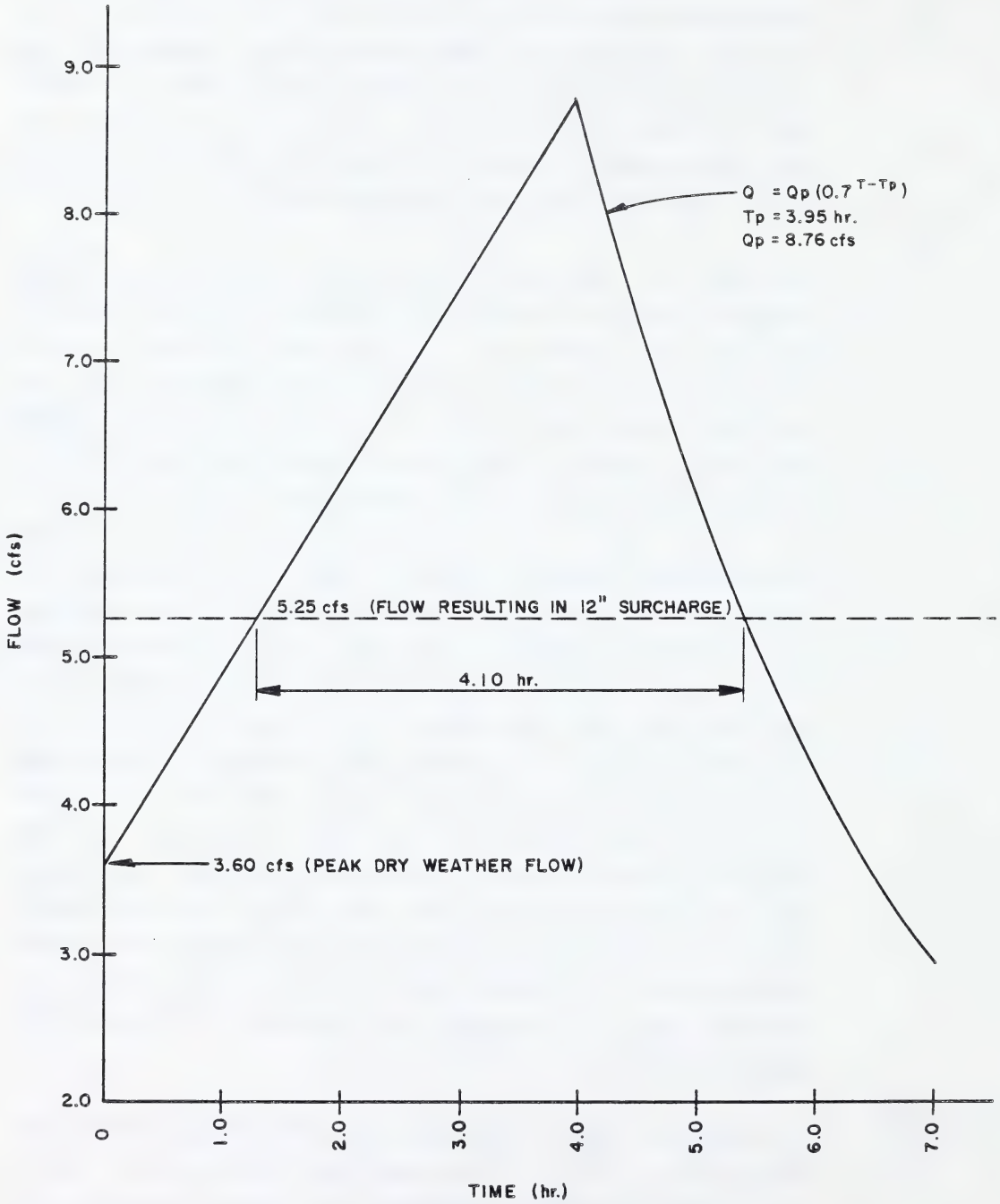


Figure 7

SANITARY UNIT HYDROGRAPH
AT MONITORING STATION No. 5

CONSEQUENTLY THE 20 ACRE SANITARY SYSTEM WILL BE DESIGNED TO PROVIDE UP TO 4.5 HOURS OF STORAGE DURING STORM EVENTS WHEN THE DOWNSTREAM SYSTEM IS SURCHARGED IN PLACES TO A DEPTH OF MORE THAN 12 INCHES.

3.3 Comparison of Theoretical with Actual Hydrograph

.1 Figure 5 on page 20 presents a sanitary wet weather hydrograph arising from a July 17, 1986 storm. This hydrograph compares quite well with the unit hydrograph in Figure 7 on page 26. The two points of comparison for the July 17th hydrograph and the unit hydrograph are Time to Peak (Tp) and k value. A discussion of these points follows:

.2 As shown on Figure 5, the Time to Peak for the July 17, 1986 storm occurs 95 minutes or 1.58 hours after the peak of the storm. This varies from our calculated value of 4.0 hours for the following reasons.

a) The July 17th storm was considered to be in the order of a 1:5 year storm, therefore, a higher intensity storm may result in a longer Time to Peak.
b) The July 17th storm was preceded by a lengthy (11 hour) "drizzle" which would reduce the effect of the actual peak.
c) The measurement of the storm occurred at a location near but outside of the drainage basin resulting in the possibility of error in the Time to Peak depending on which direction the storm was moving.

.3 The k value for the receding slope of the July 17th storm was calculated to be in the order of 0.77. We believe the difference of the k value from the estimated 0.70 resulted from the short, but quite intense rainfall between 15:15 hours and 15:35 hours. This rainfall resulted in pushing up the receding line slightly by adding additional rainfall to the system.

Overall, this "one time storm" compares well with the established unit hydrograph and substantiates the usage of the unit hydrograph for this report.

3.4 Alternative Designs

As discussed previously, sanitary sewage flow from the proposed 20 acre subdivision requires storage for a 4.5 hour period. The elevation of the 20 acre subdivision is such that a sewage lift station is required. During the conceptual design phase, various alternatives for effecting the 4.5 hour storage were considered.

.1 Determination of Start - Time for Storage

Start time for storage will take place when a rainstorm causes a 12" surcharge in the flow in the downstream system. The 12" surcharge at one manhole occurs in conjunction with various forecastable flow characteristics at other manholes. Therefore any manhole could be chosen for monitoring. However, the manhole at Station 71+94.86 was chosen because that was the manhole where the 12" surcharge would occur and a 12" surcharge is more easily identifiable and measurable than other manifestations of high flow. It was decided to provide an automatic sensor in the manhole since this was considered to be more efficient, more foolproof and more economical than having it done manually. The type of sensor chosen was the floating bag mercury switch type which has shown a high reliability in hundreds of applications for many years. The high water signal will be transmitted by telephone wire to the site of the storage facility.

.2 Method of Controlling Flow

At the commencement of the storage phase, flow from the 20 acre subdivision has to be terminated. This could either

be done manually or automatically. It was decided that the automatic method would be more efficient. Shut-off would be achieved by a motor-operated valve on the discharge line from the subdivision with operation being effected by the telephone signal from the sensor manhole at Station 71+94.86.

.3 Storage Before or After Lift Station

The required volume of storage was determined as being the cumulative peak 4.5 hour flow from the 20 acre subdivision;

$$V = 0.20 \text{ cfs} \times 0.9 \text{ (avg.factor for 4.5 hr flow)} \times 4.5 \times 60 \times 60$$
$$= 2{,}916 \text{ cu. ft.; say } 3{,}000 \text{ cubic feet.}$$

If the storage facility were to be located on the upstream side of the lift station, flow could be controlled quite simply by the operation of the pumps. However, it was considered that it would be expensive to construct a 3,000 cu.ft. "wet well" at a depth of 24 feet below ground surface. Also, a facility at a higher level could allow for a return circulating flow which would mitigate against the occurrence of septic conditions. By locating the storage facility downstream of the lift station, it could be located as high as climatic conditions or aesthetics would allow.

.4 Above or Below Ground?

From an aesthetic point of view, from a public acceptance point of view and from the point of view of economy, it was decided to locate the structure below ground.

.5 Shape of Facility

Various shapes and dimensions were considered. A circular tank concept with rotating paddles to keep the sewage in motion while being stored was considered, but abandoned when it was decided that mechanical methods could not be justified on the

basis of cost. A system of baffles and internal walls to provide efficient movement to the mass of stored sewage was also considered. However, these items could not only be expensive, but could cause future problems with respect to efficient cleaning of the facility. In the end, a rectangular shaped structure with a return flow to the lift station was decided upon.

3.5 The Preferred Design

Refer to Figure 8 on page 31 which shows a possible street and lot layout for the proposed 20 acre subdivision. The sanitary sewage storage facility will be located as shown on the plan.

A conceptual design for the proposed detention facility is shown in Figure 9, on page 32.

Normal operation of the pumping facility will consist of two pumps on an alternating cycle. Sewage will be lifted to a manhole outside the lift station whence it will flow by gravity towards the Connector Collector.

A sensor device installed downstream in the Connector Collector will send a signal to close the motorized valve whenever downstream flow surcharges to 12 inches. With the valve closed, sewage will be directed to and will be stored in the storage facility. When downstream flow diminishes, the motorized valve will open on signal and normal operations will resume.

On completion of the storage event, the walls and floors of the storage facility will be washed. This can be done by spraying either manually or mechanically by nozzles within the storage area. During the storage event, the sewage will be kept circulating by continuous closed-circuit pumping.

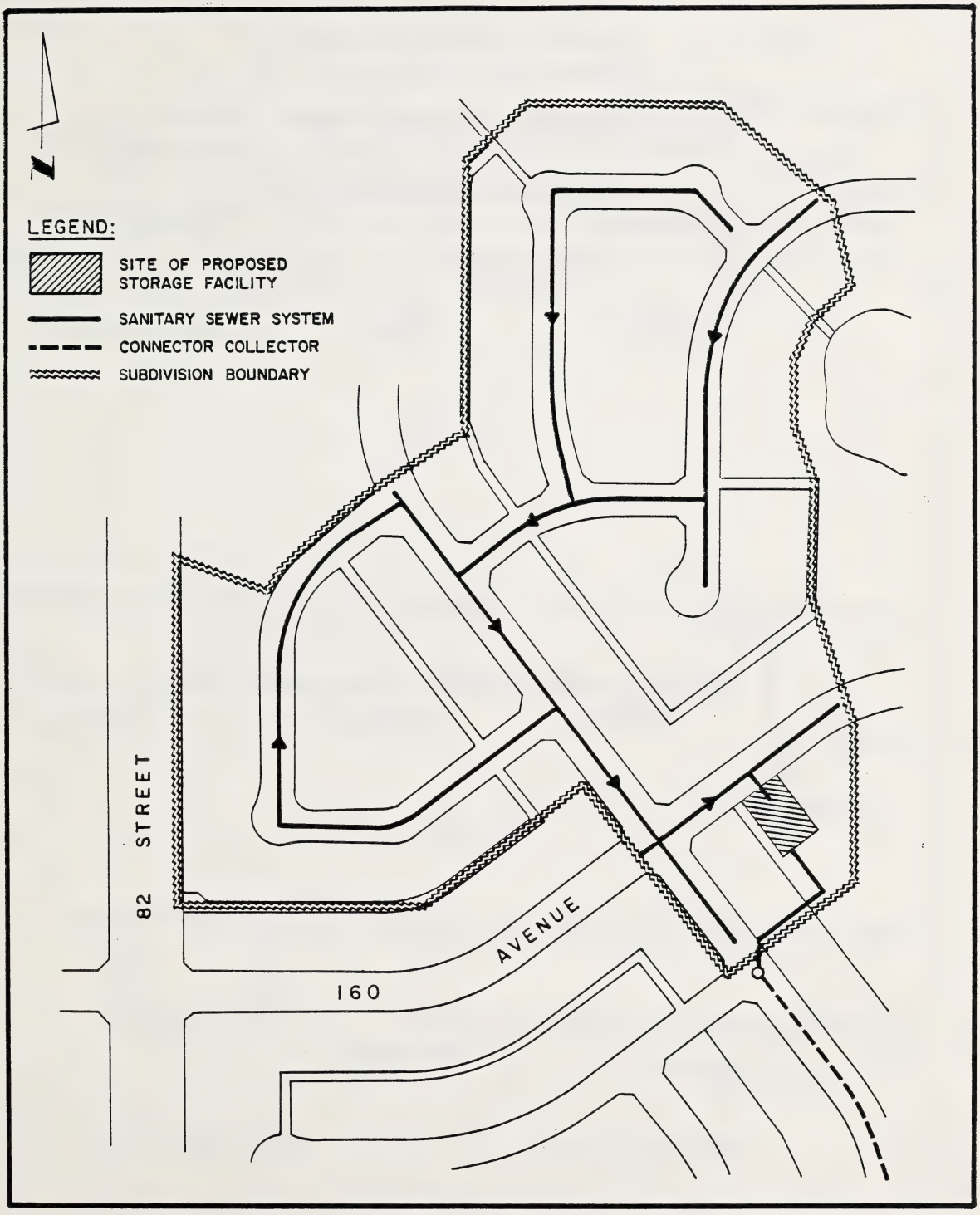


Figure 8 PROPOSED 20 ACRE SUBDIVISION; TEMPORARY SANITARY SEWAGE DETENTION SYSTEM

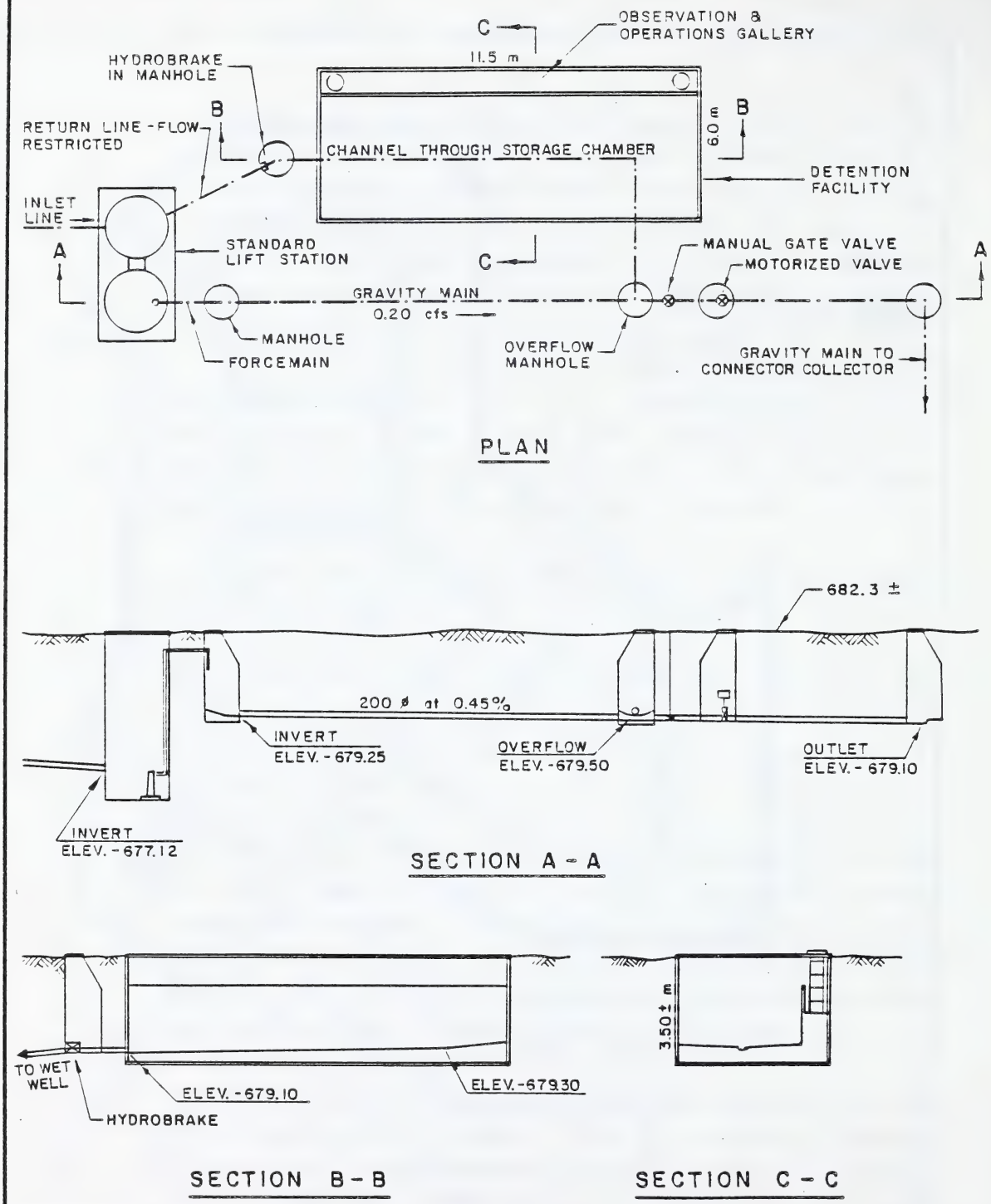


Figure 9

DETENTION FACILITY CONCEPT

Experimental exercises can be carried out at non-storm times by manually shutting down the normal operation for a period of hours. Observation and sampling capabilities would be built into the structure to enable University or other research groups to carry out experiments on peak-sharing, quality control, wash down systems, prevention of septic conditions, etc.

4.0 DETAILED ENGINEERING DESIGN AND ASSESSMENT

4.1 Detailed Design and Costing

Detailed engineering design was completed in December, 1987. The purpose of the detailed engineering design was to provide drawings and specifications for a detention facility with at least 3,000 cu.ft. of storage space.

.1 Design Elements

To service the proposed 20 acre subdivision in the Lake District in North Edmonton, the flow will be controlled by an electrically powered motor operated valve. The valve will be controlled by a sensor in the downstream manhole at Station 71+94.86.

The storage volume to be provided is 3,200 cu.ft. Peak flow over a number of hours from a population of 400 persons (assuming 60 gallons/head/day with no weeping tile connection):

$400 \times 60 \times 40 \times 0.9 \div 24$ [where 4.0 is peak factor and 0.9 is averaging factor]

= 3,620 gallons per hour

= 2,600 cu.ft. in 4.5 hours

Washdown is achieved by a row of washdown nozzles arranged about the perimeter of the facility. Water pressure will be mains pressure; about 40 pounds per square inch.

Because the volume of washdown water on a continuous basis would have exceeded the flow of sewage returning to the lift station, the washdown sequence will operate on an intermittent basis of 5 minutes per hour. It is calculated that it will take the facility five or six hours to empty after a storage event. The observation gallery and lighting will allow for observation of the system in action. Painted walls in the storage area will facilitate wash-down and cleaning.

.2 Detailed Cost Estimate

a) General & Structural Work		
General Requirements	$ 2,200	
General Contractor	50,400	
Landscaping	1,000	
Reinforcement - Supply	12,500	
Reinforcement - Install	4,300	
Miscellaneous Steel	1,100	
Painting	8,000	
Total General & Structural:		$ 79,500
b) Mechanical Work		
Water Washdown System	$ 7,100	
Ventilation System	7,900	
O & M Manuals and Drawings	1,300	
Total Mechanical:		$ 16,300
c) Electrical Work		
Distribution Equipment	$ 5,000	
Controls	5,500	
Lighting and Fixtures	5,800	
Branch Wiring	3,400	
Miscellaneous Electrical	2,100	
Total Electrical:		$ 21,800
d) Exterior Piping		
Sanitary Sewers	$ 4,000	
Manholes	14,500	
Water line	3,900	
Total Exterior Piping:		$ 22,400
TOTAL CONSTRUCTION COST:		$140,000

4.2 Cost and Performance Comparisons with Conventional Practice

As noted in section 1.1.2, constructing a conventional system would cost some $4,000,000 by 20 acres of development which could not be sustained by it. Even allowing for future cost recovery, there would be huge interest payments to be paid by subsequent developers in the Lake District/Pilot Sound Drainage Basin.

Alternatively, the downstream system could be improved by removing bottlenecks in the system. This approach would entail enlarging the pipe in about four locations at a cost of about $500,000. In conjunction with this, hundreds of thousands of dollars would probably be required to improve lot drainage and to carry out other improvements to reduce the flow of storm water to the sanitary sewer system.

The system developed and described in this report would cost less than $200,000 to construct and, as designed, would cost only about $3,000 per year to operate.

4.3 Regulatory Comments and Assessment

During the course of preparing this report, the City of Edmonton Water and Sanitation Department and Alberta Environment, Municipal Engineering Branch have been aware of the project and have provided comments and valuable advice.

The City's Water and Sanitation Department have reviewed the conceptual design and have agreed with the concepts provided therein. Moreover, they have commented as follows:

> "Further, we will review and approve the implementation of these concepts on a pilot project basis if they are properly engineered into the Sanitary scheme servicing the project area and if the operational and maintenance needs of the sewer network are adequately addressed. To gain further knowledge about the operational and mainten-

ance aspects of these concepts, we request that a continuous monitoring program with possible further assistance from Alberta Department of Housing be pursued by you as part of the pilot project activities."

The Department of Environment has reviewed the Detailed Engineering Design and has stated that the sewage detention facility will be regarded as a wet well and all standards generally required for a wet well will be applied to the proposed facility. The Department has declared, after a cursory review, that the drawings and specifications appear satisfactory, except perhaps for the proposed ventilation system.

The designed ventilation system, to be triggered when the access hatch is opened, is for 6 air changes per hour. The Department of Environment has declared that, for this intermittent system, they would prefer 30 changes per hour; or failing that, have no internal ventilation at all.

The concerns of Alberta Environment will be addressed when the design is being reviewed for construction approval by the City of Edmonton.

5.0 THE POTENTIAL OF LARGE SCALE APPLICATIONS

5.1 Status To Date

The facility design for Lake District, Neighbourhood 7 is for a 22 acre, 100 lot, 326 person development. The design allows for complete detention of peak sanitary sewage flow for 4.5 hours whenever heavy downstream flows, caused by a rainstorm event, requires it. During dry weather, the downstream system has sufficient capacity for the aforementioned 100 lot development without any detention.

5.2 Objective Of This Section

The objective of this section of the Report is to examine the feasibility of increasing the scope of the detention concept. The concept to be examined will not necessarily be tied to downstream rainstorm conditions. Rather, it will address itself in general terms to the recognition of spare capacity in apparently fully utilized sanitary sewer systems. The recognition and use of such spare capacity could increase the area of land that can be serviced by the system.

5.3 Assumptions and Parameters

To investigate the feasibility of large scale systems, certain assumptions were followed and certain design parameters were established. These are:

a) Sewer flows are determined by the Manning Formula.

b) Peak factor is derived from Harman's Formula.

c) Single family lot development is assumed.

d) Number of persons per lot is 3.26 and average contribution per person is 60 gallons/day. Weeping tile is not connected to sanitary sewer.

e) Number of lots per acre is 4.3 lots/gross acre, calculated on the basis of each lot being 55 feet x 120 feet and circulation and reserve dedication being 35% of gross area.

f) Refer to Figure 4, a hydrograph of dry weather flow, 14th of August 1986 found on page 19. From the graph the average flow is calculated as being 0.87 cfs. Also, between 2:00 a.m. and 7:00 a.m. the average flow is calculated as being 0.32, 36.5% of the average flow. For the purposes of this report, for all scenarios, the average flow during the five hours between 2 and 7 a.m. is assumed to be 50% of the daily average flow.

5.4 Discussion

A sanitary sewer is designed to accommodate the highest flow expected within a twenty four hour period. This typically happens at breakfast or supper time. During the early morning hours the flow in a sanitary sewer is generally at a low level and it is at this time that stored sewage from new development could be introduced into the system without affecting existing capacities of mains, trunks or treatment plants.

This section of the report will endeavour to determine the amount of land that can be added to an apparently fully utilized sanitary system. To add such land, sewage has to be stored, and structures and controls will be similar to those designed for the Lake District Neighbourhood 7 Sanitary Sewage Detention Facility.

5.5 Computations

The spare capacity in a sewer is taken as the volume of sewage that can be added to its normal flow between 2 a.m. and 7 a.m. so that it runs full during that five hour period.

Consider an 8" diameter sewer at 0.40 % slope

Capacity flowing full = 0.755 cfs [Manning]

Assuming that the 0.755 cfs is dry weather flow,

$$0.755 = \frac{P_f \times \text{Population} \times 80}{24 \times 3600 \times 6.24} \qquad [P_f = \text{peak factor}]$$

A sewage flow of 80 gallons/capita/day is assumed for an existing system with weeping tile connections.

The solution to the foregoing equation is:

P_f = 3.70 and the population = 1,400

Peak Flow

The average flow = 0.755 ÷ 3.70

= 0.204 cfs

From 5.3 f) the average flow between 2 a.m. and 7 a.m. is one-half of 0.204 = 0.102 cfs. Spare capacity during the five hour period is the volume of sewage computed as follows:

Capacity of Pipe — Hrs. — Min. — Sec.

(0.755 - 0.102) x 5 x 60 x 60

= 11,754 cubic feet

The area that can be added to the drainage basin by way of the spare capacity in the 8 " diameter sewer is that area that provides the following average flow,

11,754 ÷ 24 x 60 x 60

= 0.136 c.f.s.

$$\text{Average Flow} = 0.136 = \frac{\text{Population} \times 60}{24 \times 3600 \times 6.24}$$

A sewage flow 60 gallons/capita/day is assumed for a new system without weeping tile connections.

Therefore Population = 1,200

= 368 single family lots (say 370)

= 86 acres

5.6 Applications

The foregoing computation means that if sewage from an 86 acre, 370 lot, 1,200 person development is stored in a detention facility from 7 a.m. until 2 a.m., it can be released into an apparently fully utilized 8" sanitary sewer between the hours of 2 a.m. and 7 a.m. without any surcharging taking place. The volume of storage required will be 11,754 cubic feet.

To allow for storm flow restrictions downstream, add to the storage the volume of 6.0 hours peak flow. This is 33% more than was added for the Lake District Neighbourhood 7 facility. [Note that the Lake District Neighbourhood 7 is designed only for storm flow restriction.]

Required added storage = $0.135 \times P_f \times 6.0 \times 60 \times 60 \times 0.9$

P_f for 1,200 persons = 3.76

Thus, the storm flow storage = 9,870 cubic feet

Therefore total storage required = 11,754 + 9,870

= 21,624 cubic feet

Similar computations were performed for spare capacity in 10", 12" 15" and 24" diameter sanitary sewers. The results are presented in Table 5, page 42.

It can be seen from Table 5 that 86 acres of development can be added to a drainage basin by tapping the new development into an existing 8" main and providing storage for 21,624 cubic feet of sewage. 172 acres (86 x 2) could be added to a drainage basin by connecting twice to existing 8" mains. This could provide 740 single family lots.

For each 86 acre development, storage required is 11,754 cubic feet for dry weather flow and 9,870 cubic feet for wet weather flow. For a specific application, monitoring and study of the downstream system is required to determine the required storage for dry weather and wet weather flows. Note that "wet weather flow" mentioned here refers to wet weather flow in the downstream system. The new upstream system will be designed so that there is a minimum of wet weather flow allowed into the system. Weeping tile drains would not be attached to the sanitary system.

The detention facility required for an 86 acre development would be required to store 21,624 cubic feet (say). Unlike the Lake District Neighbourhood 7 facility, this facility would have no

T A B L E 5

SPARE CAPACITIES IN SANITARY SEWERS

Dia. & Slope of Downstream Sewer	Capacity c.f.s.	Populat'n Served	Spare Capacity cu.ft.	Spare Capacity Can Service			% Increase In Popul'n	Storm Storage Requ'd	Total Storage Requ'd
				Pop.	Lots	Acre			
8" @ 0.40 %	0.76	1,400	11,754	1,200	370	86	86%	9,870	21,624
10" @ 0.30 %	1.20	2,400	18,440	1,920	590	137	80%	14,900	33,340
12" @ 0.20 %	1.60	3,100	24,400	2,540	780	180	82%	19,270	43,670
15" @ 0.15 %	2.53	5,000	38,150	3,970	1220	280	79%	28,640	66,790
24" @ 0.14%	8.50	21,860	123,800	12,875	3950	920	59%	79,000	202,800

observation gallery. Therefore, depth of storage could be 3.0 metres instead of 1.5 metres. A 20m x 10m x 4.5 metre deep structure could store 21,200 cubic feet of sewage.

The 21,000 cubic feet facility would have the same external piping layout and the same washdown system as the Lake District Neighbourhood 7 facility. The controls would be similar. However, instead of confining flows strictly to the 2:00 a.m. to 7:00 a.m. time slot, the system would work on releasing sewage at whatever time of day the downstream flow would allow. This would speed up the release of stored sewage and reduce the danger of the occurrence of septic conditions.

The storage required for wet weather flow will be required only on an intermittent basis as storms affect the downstream system, and only when the effect of these storms occurs between 2 and 7 a.m.

For any specific application of sanitary sewage detention as outlined above, it is recommended that design be preceded by at least 6 months of monitoring, preferably April to September inclusive.

5.7 Construction Cost Estimate - 21,000 cubic foot Facility

.1 General & Structural Work

General Requirements	$ 2,500	
General Contractor	126,000	
Landscaping	1,000	
Reinforcing + Miscellaneous Steel	45,000	
Painting	-	
Total General & Structural:		$ 174,500

.2 Mechancial Work

Water Washdown System	$ 12,000	
Ventilation System	-	
O & S Manuals and Drawings	1,300	
Total Mechanical:		$ 13,300

.3 Electrical Work

Distribution Equipment	$ 5,000	
Controls	5,500	
Lighting and Fixtures	-	
Branch Wiring	3,400	
Miscellaneous Electrical	2,100	
Total Electrial:		$ 16,000

.4 External Piping

Sanitary Sewer	$ 4,000	
Manholes	14,500	
Water Line	3,900	
Total External Piping:		$ 22,400
TOTAL CONSTRUCTION COST:		$ 226,200

Note that from the above estimate we have deleted

a) Painting - probably not required on a working system,

b) Ventilation - to be manual, as required,

c) Lighting - not required.

The facility will probably operate in conjunction with a pumping station unless topography allows otherwise. Pumping station costs are not included as they are typically required by other servicing design considerations (i.e. not necessitated because of the storage facility).

The cost per acre, for an 86 acre extension is $226,200 ÷ 86 = $2,630/acre or $6,500 per hectare. Add 15% for Overhead and Engineering, and the cost per hectare equals $7,475.

For an extension to an Edmonton drainage basin, this $7,475 per hectare would substitute for the "permanent area contribution" (p.a.c.) presently levied in the City of Edmonton for sanitary trunk servicing. 1987 p.a.c. rates in Edmonton, for basins with more than 80 acres still to develop average $8,570 per hectare.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary of the Main Findings

.1 It is possible to economically construct a sanitary storage facility for a 20 acre development in the Lake District, Edmonton to protect the downstream system during heavy storm events.

.2 The concept can be expanded to increase the size of sanitary collection basins by more than 50% at costs considerably below those for conventional methods. Moreover, there is no reduction in service levels with the new concept.

6.2 Recommendations

.1 It is recommended that the storage facility proposed for the 20 acre development in the Lake District be constructed in conjunction with the subdivision.

.2 It is further recommended that the Lake District storage facility be operated as a pilot project, so that operating, maintenance and cost information can be documented and utilized for the planning of future facilities.

.3 It is recommended that concepts developed herein, for expanding the size of drainage basins, be applied to locations in Alberta where such expansion may be beneficial.

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